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In-Situ Undrained Shear Strengths and Permeabilities Derived from Piezometer Measurements

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ABSTRACT: Existing theories and models describing stress changes and consolidation-time effects around a pile were used to derive in-situ permeabilities and undrained shear strengths from piezometer probe measurements in smectite- and illite-rich soils. Permeabilities derived from piezometer measurements are in reasonable agreement with laboratory measurements, and calculated undrained shear strengths agree well with strength measurements using standard field and laboratory techniques.

Undrained shear strengths S_u were estimated using insertion pressures U_i, determined from both the 10.2- and 0.8-cm-diameter probes, and the relationship, $U_i = 6 \times S_u$. A strength measurement determined with the small diameter probe inserted in the disturbed zone of a previously emplaced 2.5-cm-diameter cylinder showed a significant strength reduction equal to half the value determined for the soil (strength) in the zone unaffected by the implanted cylinder. Differences in decay rates were significant, indicating severe soil disturbance in close proximity to the cylinder.

Multisensor piezometer probes (2), 10.2 cm in diameter, were deployed in shallow-water fine-grained smectite-rich soils of the Mississippi delta. Pore-water pressures were measured at subbottom depths of 6.5, 12.6, and 15.6 m. Insertion pressures, time-dependent pore pressure decay, and ambient excess pore pressures were determined.

Single sensor piezometers (2), 0.8 cm in diameter, were developed for deep-ocean investigations. Before high pressure testing (55 MPa), probes were inserted in reconstituted illitic marine soil to depths of 16.9 and 26.4 cm below the soil-water interface. Insertion pressures and their decay characteristics were monitored.

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Significant differences were observed in the pore-pressure decay rates produced by the small and large diameter probes. Decay times for the induced pressures to reach t_{100} values were on the order of tens of hours for the large diameter probes, whereas the t_{100} values of the small diameter probes were on the order of minutes. These differences in decay times were a function of the differences in probe diameters (radii) and soil permeabilities.

KEY WORDS: piezometers, pore pressures, shear strength, permeability, heat transfer, marine geology, piezometer probes, consolidation, mass physical properties

Existing models describing stress changes and consolidation-time effects around a pile driven into fine-grained cohesive soils and limited field and laboratory measurements are used to derive in-situ undrained shear strengths and permeabilities of selected coastal and deep-ocean marine soils. Theory and models of interest in this study were developed by Soderberg [*1*], Esrig et al [2], Randolph et al [*3*], and Wroth et al [*4*] and will be discussed later in limited context as applied to the derivation of selected geotechnical parameters. Piezometer instrumentation and field and laboratory investigations were discussed in detail elsewhere and will be reviewed briefly here for background information only [*5–7*].

Pore-pressure data used in this study are taken from piezometer investigations at two offshore sites in the Mississippi delta (East Bay and Main Pass areas) and from a laboratory study of reconstituted illitic "red" clay from the Pacific deep-sea basin [*6,8,9*]. A general description of the Mississippi delta and Pacific basin soil properties is found in Tables 1 and 2.

Background

Shallow-water and deep-ocean piezometer probes were developed to provide in-situ data on pore-water pressures and related geotechnical properties of submarine soils [*10,11*]. The purposes of the shallow-water piezometer investigations were to (1) determine the feasibility of making long-term pore-pressure measurements in the ocean environment; (2) assess ambient and dynamic pore pressures in selected submarine soils; (3) determine insertion pore pressures (U_i max) and their decay characteristics; and (4) assess the vertical effective stress (state of stress) and stability of deposits at selected sites in the Mississippi delta [*6*].

The purpose of developing the deep-ocean piezometer probe was to investigate in-situ pore-water pressure measurements at nominal ocean depths of 6000 m and at seafloor subbottom depths of 1.0 m [*11*] in support of the U.S. Sandia National Laboratories Subseabed Disposal Program [*12*]. The technology would then be extended to make pore-pressure measurements to much greater seafloor subbottom depths (tens of metres) for addressing numerous relevant engineering and geological questions and problems. Examples would be questions and problems relating to offshore engineering construction (pipelines, platforms, and settlement), and geological processes (seafloor stability, mass movement, liq-

TABLE 1 — *Geotechnical data: Mississippi delta.*

Site, Units	Shear Strength (Field), kPa	Shear Strength (Lab), kPa	Water Content, %	Liquid Limit, %	Plastic Limit, %	Wet Unit Weight, Mg/m^3	Specific Gravity, G_s	Sand Silt Clay, %	Soil Description
					1977				
Block 28	2 to 5	~3.0	95 to 110	65 to 85	30 (average)	1.46 to 1.50	2.70	1 sand 25 silt 74 clay	Silty clay average <1% sand, worm burrows faint laminations, gas bubbles apparent
					1978				
Block 73	6 to 23	~6.8 (average from four cores)	25[a] to 120	60 to 90	40	1.56 (average from four cores)	2.70 to 2.78	10 sand 25 silt 65 clay	Silty clay average <5% sand (81% sand maximum), sand lenses and laminations.

NOTE: Data from Ref *6*.
[a]Low value due to high sand content.

TABLE 2 — *Geotechnical data: ISHTE simulation.*

Soil Depth, cm	Shear Strength Vane Probe, kPa	Water Content, %	Liquid Limit, %	Plastic Limit, %	Wet Unit Weight, Mg/m^3	Specific Gravity, G_s	Sand Silt Clay, %
16.9	2.0 to 3.6 2.2 average	101 average	77 average	34 average	1.49	2.77	0 sand 33 silt 67 clay
26.4	1.8 to 4.0 2.6 average	same	same	same	1.49	same	...

NOTE: Data from Ref *18*.

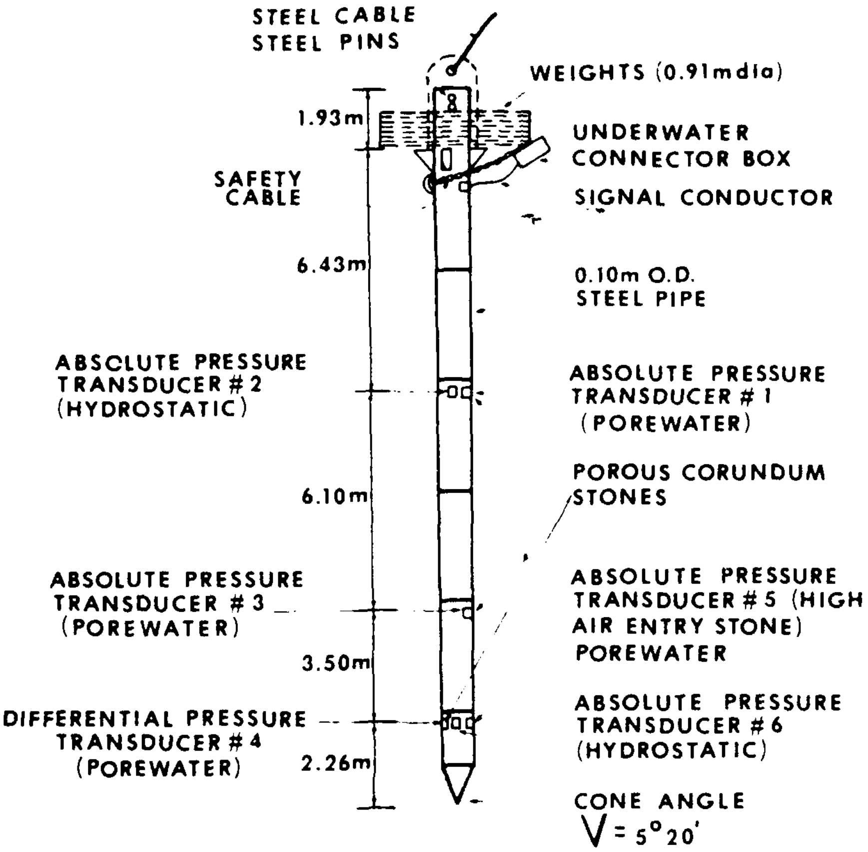

FIG. 1—*Shallow-water piezometer (multisensor) probe (10.2 cm diameter) deployed in East Bay and Main Pass, Mississippi delta sites. Diagram depicts position of sensors along the probe length.*

uefaction potential, and consolidation/compaction [lithification]) relating to transport, deposition and diagenesis of fine-grained soils.

Development of the deep-ocean piezometer probe has been an integral part of Sandia National Laboratories' U.S. Subseabed Disposal Program (SDP) and their In Situ Heat Transfer Experiment (ISHTE). The SDP is studying the feasibility of emplacing high-level nuclear waste in fine-grained deep-sea formations of the world ocean basins [*12*]. ISHTE is designed to provide data for the comparison of in-situ thermal response of marine soil with numerical predictions. These analyses will be used to evaluate the feasibility of subseabed disposal [*8,13*].

Instrumentation

The shallow-water multisensor piezometer probe consists of 0.10-m-diameter seamless steel pipe segments 3.05 m in length which, are connected with O ring sealed couplings (Fig. 1). Variable reluctance type pressure transducers are enclosed in capsules within the probe shell in close proximity to a porous filter. This design permits selection of the sensor and porous filter position along the probe length. Filters are coarse corundum and high air entry stones having approximate

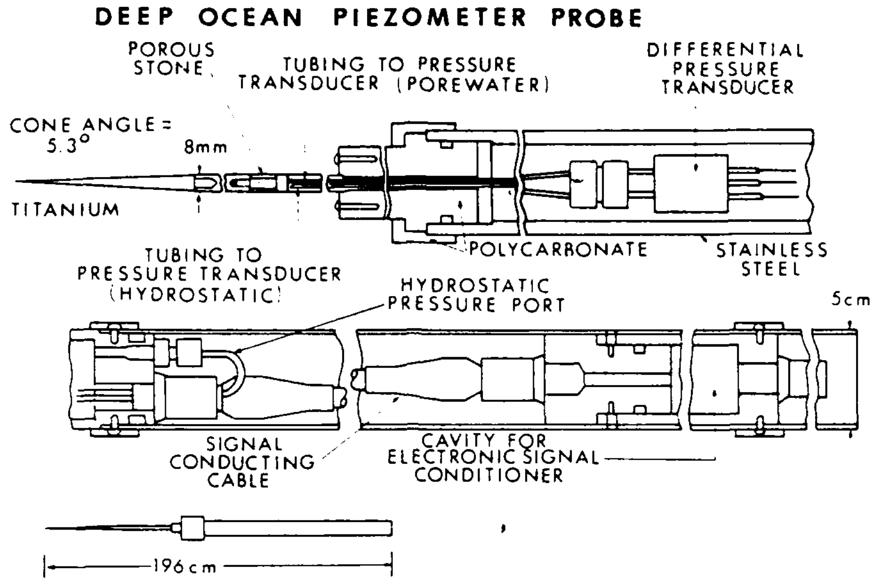

FIG. 2—*General mechanical configuration of the deep-ocean piezometer probe developed for the in-situ heat transfer experiment (ISHTE).*

porosity n and permeability k values of n = 45 to 50%, k = 0.1 to 0.3 mm/s and n = 35 to 38%, k = 1 to 3 nm/s, respectively. The reason for the use of different filters was discussed by Bennett and Faris [*10*]. The probe tip (~5° cone angle) was designed to control sediment disturbance and to produce plane-strain soil deformation during probe insertion (Fig. 1). Using this tip design (~5° cone) and a L/D ratio of ≥20 from tip to filter, the induced pore pressures (U_i) generated during probe insertion can be related directly to the undrained shear strength of the soil [*2,10*]. Also, additional geotechnical parameters can be derived, with confidence, from the pore-pressure data as will be discussed later. Details of the shallow-water piezometer probe can be found elsewhere [*6,7*].

The deep-ocean, single-sensor piezometer probe utilizes the same cone design but consists of a titanium tube of only 8 mm diameter. A single porous stone (corundum) placed ~16.0 cm above the probe tip allows pore-water pressure to be transmitted to a variable reluctance differential pressure transducer (Fig. 2). The pressure transducer operates at ambient hydrostatic pressure (~69 MPa). Titanium is used for the tip and 8-mm tube because of the severe environmental conditions that exist during the experiment: (1) high hydrostatic pressure (~69 MPa); (2) high temperature differential along the length of the probe (ΔT ~ 296°C; range ~4 to 300°C); (3) strong electrolite in contact with the probe (~35 parts per thousand salinity); and (4) potential for thermogalvanic corrosion. Details of the deep-ocean piezometer probe have been discussed by Bennett et al [*6,7,11*].

The deep-ocean piezometer data discussed in this paper were obtained during a laboratory (simulation) experiment conducted by Sandia National Laboratories and participating institutions [*8*]. This experiment was designed to test some of

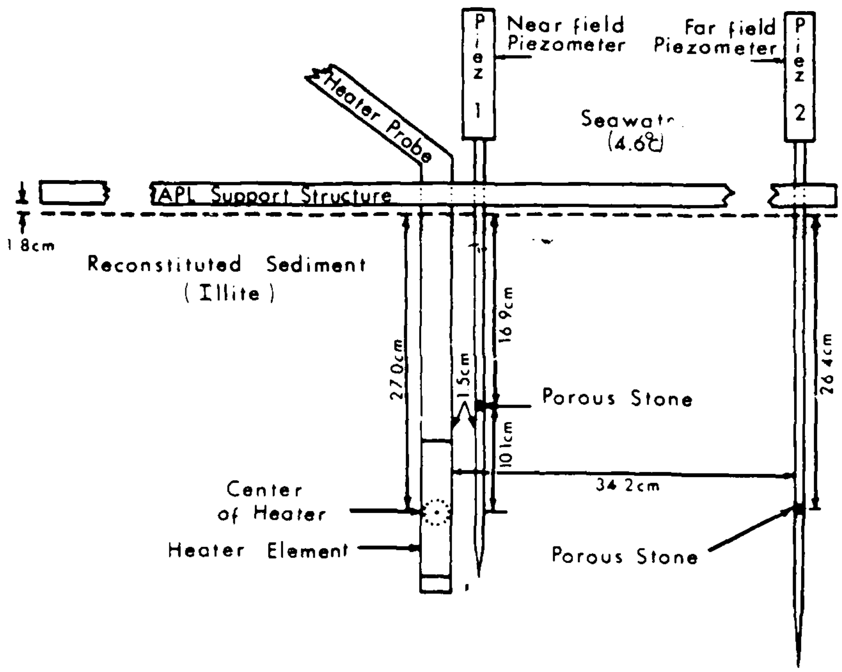

FIG. 3—*Deep-ocean piezometer probes (8 mm diameter) in reconstituted illitic "red clay" soil during a high pressure (55 MPa) simulation test. Diagram depicts position of piezometers in relationship to heater probe.*

the instrumentation and components that will eventually be part of ISHTE. The experiment consisted of a scaled test designed to simulate environmental conditions predicted for the field experiment [*8*] and consisted of a cylindrical container of about 1 m^3 of remolded, reconsolidated, illite clay soil [*9*]. Instrumentation was fitted to a support frame over the container of soil and placed in a hyperbaric chamber capable of maintaining 55 MPa at 4°C for a period of approximately one month [*14*]. Piezometer probes (No. 1: near field, and No. 2: far field) were inserted in the test tank before pressurization of the chamber at positions of 1.5 and 34.2 cm from a previously implanted heater probe (Fig. 3). Details of the experiment and instrumentation developed by the various participating institutions are discussed elsewhere [*7–9,14*].

General Theoretical Considerations

During the insertion of cylindrical pipes (piles and probes) in normally and underconsolidated cohesive fine-grained soil, significant induced excess pore pressures (U_i) are generated. The time required for these excess pressures to dissipate to ambient pressure is, to a first approximation, a function of the probe (pipe) radius and the soil coefficient of consolidation, which depends upon the permeability of the material. The governing equation for the excess pore pressure for radial consolidation (cylindrical cavity expansion) is expressed as

$$\partial u/\partial t = C_v\{(1/r)(\partial/\partial r)[r(\partial u/\partial r)]\} \qquad (1)$$

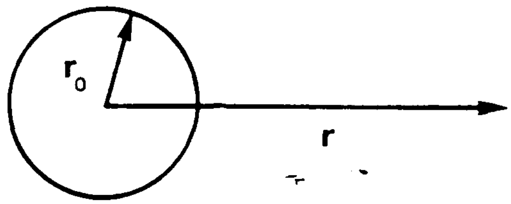

FIG. 4—*Coordinate system used in the solution of the equation for radial consolidation (cylindrical cavity expansion) for a probe (or pile) of radius* r_o *and pore pressure distribution to radial distances* r.

where u is the excess pore-water pressure, C_v is the coefficient of consolidation, t is the time, and r is the radial coordinate (Fig. 4) [*3*].

The following assumptions are made in deriving Eq 1:

1. The medium (here, marine soil) is assumed to be a porous elastic matrix containing a viscous fluid (seawater).
2. Pore fluid flow can be related to the pore pressure by Darcy's law.
3. The consolidation process assumes, primarily, radial flow of the pore fluid and radial expansion of the elastic matrix.
4. Thermal effects are not considered in the formulation.

The solution of Eq 1 for a piezometer probe (or pipe) of radius r_o (Fig. 4) can be obtained if the initial pore fluid pressure distribution and the coefficient of consolidation are known. For radial expansion C_h is considered $\cong C_v$ and is a reasonable assumption for surficial submarine soils having a random soil fabric [*15*].

Soderberg [*1*] obtained two solutions for the initial pore fluid pressure distributions from the pile skin r_o to radial distances r assuming the soil is (1) an elastic-plastic material and (2) a viscous material (Fig. 5). The most important assumption in deriving the pore-pressure distribution, analytically, is that the excess pore pressure generated during probe insertion is proportional to the imposed radial stresses [*1*]. Given the initial pore fluid pressure distributions for the two assumed plain stress soil systems (elastic-plastic and viscous), Soderberg

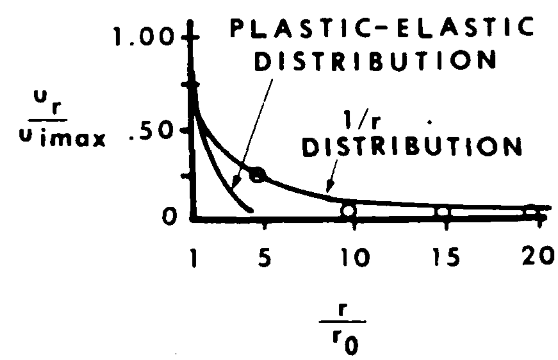

FIG. 5—*Two solutions for the initial pore pressure distributions caused by cylindrical cavity expansion of a soil mass as a function of distance* r/r_o *from the probe (pile) skin* [1].

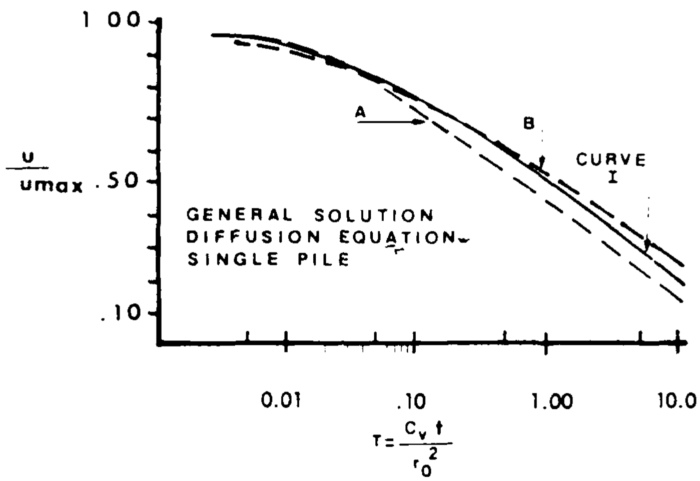

FIG. 6—*Pore pressure as a function of time (dimensionless factor* $T = C_v t/r_o^2$*) at the pile skin. Curve A is for elastic-plastic solution; Curve B is for a viscous assumption; mean distribution is depicted on Curve I. (Ref. 1).*

[*1*] obtained solutions for the pore-pressure dissipation at a pile (probe) skin (at r_o) and derived a general solution (curve) that lies approximately between the two initial solutions (Fig. 6).

Curve A (Fig. 6) depicts the pore pressure as a function of time $T = C_v t/r_o^2$ at the probe surface in terms of the initial pore pressure $U_{max} = U_i$ at the probe surface for an elastic-plastic soil. The pore pressure variation is shown as Curve B for the viscous assumption. The solution for the mean distribution is depicted as Curve I.

Solutions for the decay of pore pressure at a probe (pile) surface also have been obtained by Wroth et al [*4*] in a form which plots U/U_{max} as a function of time $T = C_v t/r_o^2$ and with $U_{max}/S_u(o)$ as a parameter where $S_u(o)$ is the initial value of the plane strain undrained shear strength (Fig. 7). In this solution, the coefficient of consolidation is expressed as

$$C_v = [k\ 2G(1 - \nu')]/[\gamma_w(1 - 2\nu')] \quad (2)$$

where

k = the coefficient of permeability,
γ_w = the unit weight of water,
G = the elastic shear modulus of the soil, and
ν' = Poisson's ratio (drained) of the elastic soil.

With solutions obtained from Figs. 6 and 7 and the expression of C_v in Eq 2, the coefficient of permeability can be estimated from the pore-pressure measurement obtained with piezometer probes. Based on a soil modeled as an elastic perfectly plastic material, the maximum excess pore pressure at the probe surface is given by Randolph et al [*3*] as

$$U_{i\,max} = S_u \ln(G/S_u) \quad (3)$$

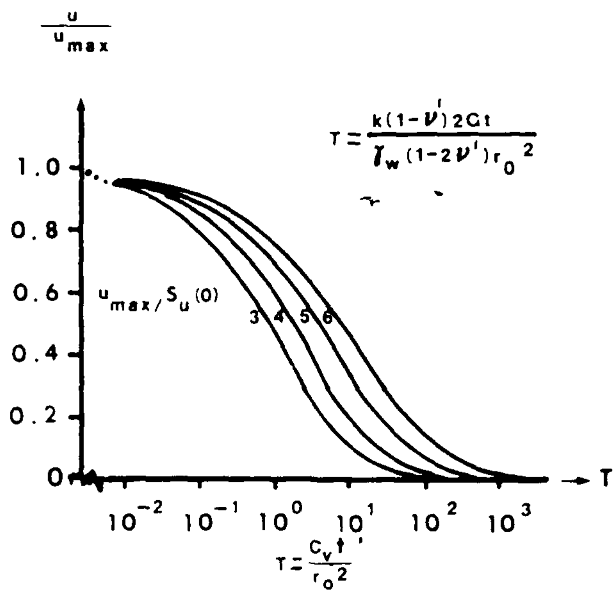

FIG. 7—*Solutions for pore pressure as a function of time at the pile skin for soils having different values for the parameters* $U_{max}/S_u(o)$ [4].

Since $U_{i\,max}$ is obtained from piezometer probe measurements and $S_u(o)$ can be determined in the laboratory, or from in-situ vane probe tests, the shear modulus G is easily calculated from Eq 3. Esrig et al [2] suggest the relationship

$$U_i = 6S_u \tag{4}$$

for lean inorganic soils of moderate to high sensitivity. The value of six (6) is considered reasonable since studies indicate that the predicted changes in radial total stress and pore pressure are relatively insensitive to the soil modulus ratio (E_u/S_u), where E_u is the soil modulus and S_u is the undrained shear strength [2]. Given a value of $U_{i\,max}$ (insertion pressure) obtained from piezometer measurements, the undrained shear strength can be calculated using either Eq 3 (if the shear modulus is known) or Eq 4.

The nondimensional time unit T_{50} can be determined from Figs. 6 or 7 for 50% consolidation, and t_{50} can be obtained from the piezometer probe measurements from normalized plots depicting the time-related dissipation of induced pore pressures [10]. The coefficient of consolidation can be determined by

$$C_h \cong C_v = T_{50}r_o^2/t_{50} \tag{5}$$

Thus the coefficient of permeability can be estimated assuming a reasonable value for Poisson's ratio (drained).

It should be noted that the assumption of a soil modeled as an elastic perfectly plastic material can be modified [3], and if the shear modulus G is known for the medium, Eq 3 is not required for the permeability estimate.

In-Situ Shear Strength and Permeability

Field and laboratory piezometer data were analyzed to obtain estimates of the in-situ undrained shear strength S_u, permeability k, and shear modulus G using the above relationships for the prediction of pore fluid pressure generation, dissipation, and soil strength for piles driven into cohesive soil [1–4]. Piezometer observations for the Mississippi delta and ISHTE simulation tests are found in Tables 3 and 4. The time-dependent dissipation of pore pressure for the 5.1- and 0.4-cm radius probes are depicted in Figs. 8 through 10. Using relationship $U_i = 6S_u$ for the determination of undrained shear strength suggested by Esrig et al [2], values were calculated for Mississippi delta and ISIMU simulation soils. These values are compared with both in-situ vane probe measurements and laboratory miniature vane tests (Tables 5 and 6). Vane shear probe measurements for the ISIMU soil have been discussed by Babb and Silva [16] and Silva et al [9]. Strength measurements for the Mississippi delta soils are summarized by Bennett et al [6].

Reasonably good agreement is observed between the calculated and measured undrained shear strengths for both soil types (Tables 5 and 6). The differences observed, however, in the induced pore pressures and their respective decay characteristics for the ISHTE piezometers suggest significant differences in the geotechnical properties of the soil in proximity to the two probes. The significantly lower induced pore pressure of Probe 1 and its rapid decay compared with observations from Probe 2 (Fig. 10) indicates (1) a reduction of calculated soil strength (approximately 51% less at the near field probe compared with the far field data, Table 6 and (2) a significantly shorter drainage path (at the near field probe) for the induced pore pressures to dissipate, compared with the far field piezometer. The time delay of induced pressure before dissipation at the far field probe also supports these conclusions. The severe cracking of the soil observed during heater insertion was probably a major factor responsible for the observed differences in the induced pore-pressure characteristics [17]. Vane shear measurements were determined at distances of 19 to 21 cm from the heater, and the values reflected in Table 6 were selected from the strength profile [18] corresponding to depths approximately equal to depths of piezometer measurements. Vane tests were performed about 20 h following heater insertion, thus the influence of pore pressures generated from heater insertion were not of significance. Also, vane measurements were sufficiently distant from the heater to eliminate the influence of pore pressures caused by the heater. Using Eq 3 for the estimation of U_i for the ISHTE simulation soils and comparison with pore-pressure measurements indicated agreement to within $\pm 1\%$ of the observed data (David McTigue, personal communication, 1983). These data indicate reasonably good agreement between the predictive capabilities of the models and observed measurements. Knowing U_i and S_u from piezometer and vane shear measurements, the shear modulus G was calculated (Tables 5 through 7) for use as an input parameter to Eq 2. Assuming a drained Poisson ratio of $\nu' = 0.3$, coefficients

TABLE 3 — *Piezometer observations: Mississippi delta.*

Site	Latitude Longitude	Water Depth, m	Length of Data Recorded, h	Sensor Depth Below Mudline, m[a]	Porous Filter: Corundum	Porous Filter: High Air Entry	Sensor Type: Absolute	Sensor Type: Differential	Induced Insertion Pressure, kPa (psi)	Ambient Excess Pore Pressure kPa (psi)
					1977					
Block 28	29°01 N 89°15 W	13	2650	6.5	X		X		35.1 (5.1)	4.8 (0.7)
				12.6	X		X		57.9 (8.4)	19.3 (2.8)
				15.6	X			X	55.1 (8.0)	45.5 (6.6)
				15.6		X	X		74.4 (10.8)	12.4 (1.8)
					1978					
Block 73	29°15 N 88°55 W	44	530	6.5	X		X		51.0 (7.4)	9.0 (1.3)[b]
				12.6	X			X	99.2 (14.4)	15.8 (2.3)
				15.6		X	X		107.5 (15.6)	17.9 (2.6)
				15.6	X			X	122.0 (17.7)	14.5 (2.1)

NOTE: Data from Ref 6.
[a] Estimates based on diver observations and mud remaining on weight stand.
[b] Estimates based on graphical extrapolation of decay curves.

TABLE 4 — *Piezometer observations: ISHTE simulation.*

Probe	Water Depth, m	Length of Data Recorded, min	Sensor Depth[a] Below Mudline, cm	Porous Filter	Induced Insertion Pressure, kPa (psi)	
Piezometer No. 1 (near field)	~1	75	16.9	corundum	6.6	0.96
Piezometer No. 2 (far field)	~1	300	26.4	corundum	12.9	1.88

[a] Differential pressure transducers.

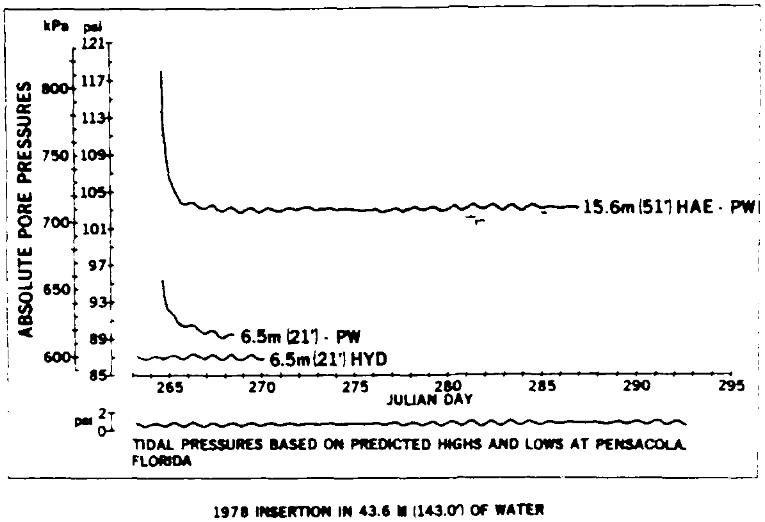

FIG. 8—*Dissipation of induced pore pressures of 10.2 cm diameter multisensor shallow-water piezometer, measured by absolute pressure sensors. Note correlation of absolute pressures with tidal pressures. Main Pass, Mississippi delta* [6].

of consolidation $C_v \cong C_h$ were calculated from Eq 2 for each soil type using piezometer data (induced pore-pressure decay, normalized with respect to U_{max}, as a function of the logarithm of time t) (Figs. 11 and 12). The derived values for the coefficient of consolidation and permeability for the 5.1-cm radius and

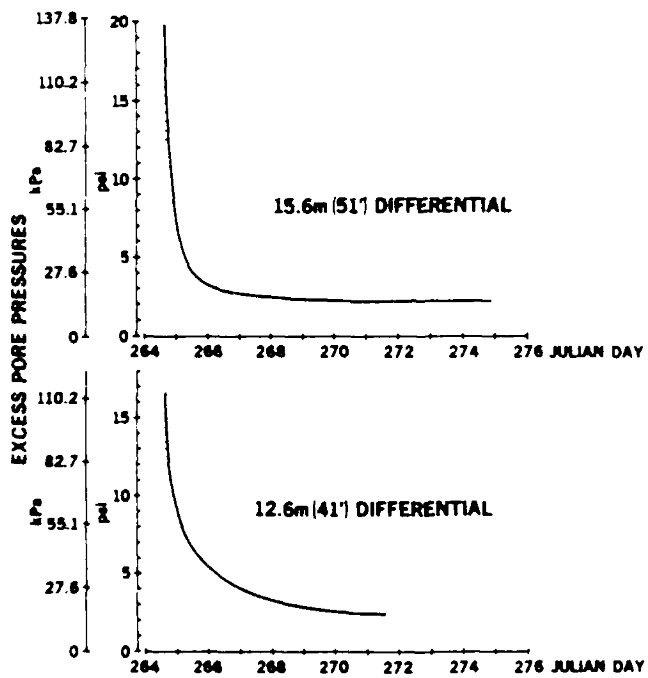

FIG. 9—*Dissipation of excess pore pressures caused by 10.2-cm-diameter probe inserted into fine-grained smectite-rich Mississippi delta soil, measured by differential pressure sensors* [6].

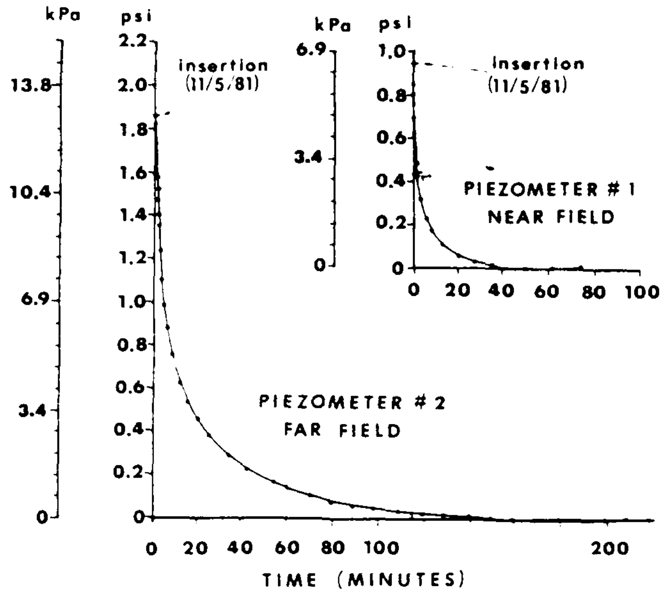

FIG. 10—*Dissipation of excess pore pressures from 8-mm-diameter probe inserted into reconstituted illitic "red clay" soil, measured by differential pressure sensors* [7].

0.4-cm radius probes are given in Table 7 (compare with Table 8) using both models depicted in Figs. 6 and 7.

Conclusions

Derived values for undrained shear strength using piezometer insertion pressures and the models are in reasonable agreement with field and laboratory measurements (Tables 5 and 6). Additional tests on various soil types would enable refinement of the solutions and models.

The estimated values of the coefficients of consolidation and permeability are in the same range of laboratory results (Tables 7 and 8) for tests performed on Mississippi delta and ISHTE simulation soil samples [*9,22*]. It should be noted that the estimated values for k and C_h are dependent on the solutions of the consolidation equation (such as Figs. 6 and 7). To a first approximation, reasonable agreement is indicated between the models and the observed piezometer measurements. Additional research of the material properties (various soil types) and more piezometer measurements are required in order to refine the analytical solution of the consolidation models.

Equation 1 would undoubtedly require modification for consolidation problems involving significant heat flow or a partially saturated soil (gas-liquid mixture). Additional models and equations would need to be developed to solve similar problems regarding estimates of undrained shear strength, permeabilities, and related geotechnical parameters.

TABLE 5 — *Calculated versus measured shear strengths: Mississippi Delta.*[a]

Pressure Sensor Depth Below Mudline, m	U_t, kPa	S_u (calculated) $U_t/6$, kPa	S_u (measured), kPa	Depth Below Mudline, m	Test Type
		BLOCK 73			
6.5	51.0	8.5	9.4	5.9-7	laboratory vane on six cores
12.6	99.3	16.6	20.0	12.4	in situ vane
15.6	107.5	17.9	24.1	15.5	in-situ vane
15.6	122.0	20.3	24.1	15.5	in-situ vane
		BLOCK 28			
6.5	35.2	5.9	5.8	5-7	in-situ vane (average of 3)
12.6	57.9	9.7	9.0 to 11.7	12.6	in-situ vane
15.6	55.2	9.2	9.0 to 11.7	15.6	in-situ vane
15.6	74.5	12.4	9.0 to 11.7	15.6	in-situ vane
			7.9 to 9.5		lab vane

NOTE: S_u = undrained shear strength. $U_t/6$ = calculated undrained shear strength.
[a] Data from Ref *6*.

TABLE 6 — *Calculated versus measured shear strengths: ISHTE simulation.*

Piezometer	Depth of Porous Stone Below Mudline, cm	U_t, kPa	S_u (calculated) $U_t/6$, kPa	S_u (measured) kPa (psi)	Depth Below Mudline, cm	Test Type
No. 1	16.9	6.6	1.1	2.2 (0.33)	17	in-situ vane
No. 2	26.4	13.0	2.2	2.6 (0.38)	26.5	in-situ vane

NOTE: S_u = undrained shear strength. $U_t/6$ = calculated undrained shear strength.
[a] Vane measurements determined 20 hours after heater insertion and 19 to 21 cm from heater (pretest unheated, Ref *18*).

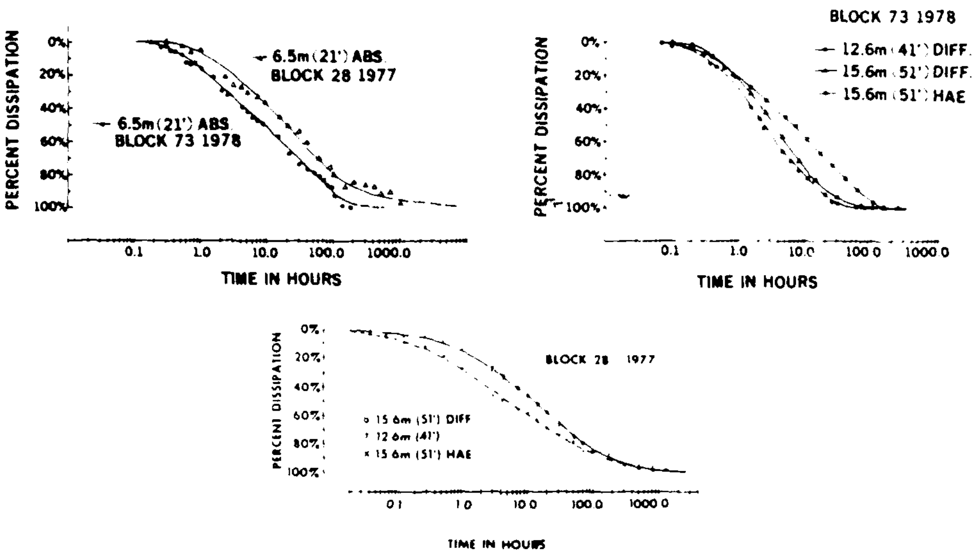

FIG. 11—*Time-related dissipation (percent dissipation) of induced (excess) pore pressures normalized with respect to* U *max for different sensors, plotted as a function of the logarithm of time. (Note different decay time for Block 28 and Block 73 soils.) (From Ref* 6*)*

Acknowledgments

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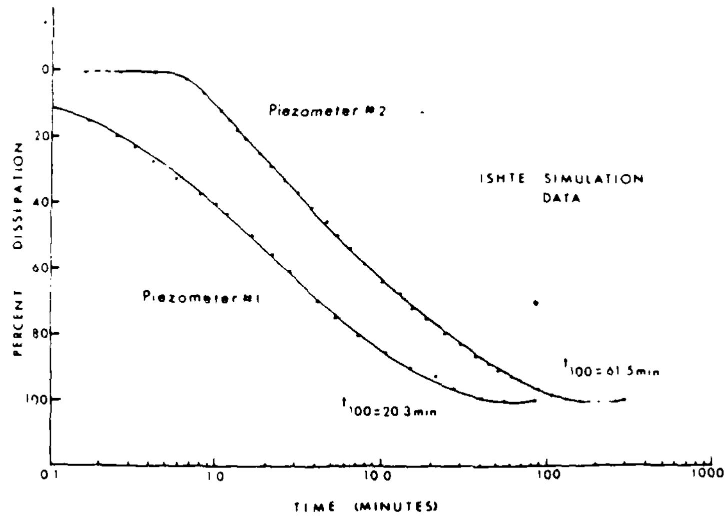

FIG. 12—*Time-dependent dissipation of induced pore pressures* (U, *max) as a function of the logarithm of time for the 8-mm-diameter probes. Note different decay times for the near field (No. 1) and far field (No. 2) probes. (From Ref* 6*)*

TABLE 7—*Estimate of the coefficient of permeability* k *and coefficient of consolidation* C_h.

Site	Pressure Sensor Depth Below Mudline, m	Shear Modulus/Undrained Shear Strength, $G/S_u(o)$	Estimated Values			
			Based on Fig. 7		Based on Fig. 6	
			C_h,[a] cm²/s	k, cm/s	C_h, cm²/s	k, cm/s
5.1-CM RADIUS PIEZOMETER PROBE (MEASUREMENTS FROM REF 6) GULF OF MEXICO						
Block 73	6.5	230	3.46×10^{-3}	4.60×10^{-8}	0.68×10^{-3}	0.91×10^{-8}
	12.6	143	3.58×10^{-3}	3.51×10^{-8}	0.99×10^{-3}	1.00×10^{-8}
	15.6	86	6.02×10^{-3}	8.43×10^{-8}	1.98×10^{-3}	2.75×10^{-8}
	15.6	156	11.3×10^{-3}	8.72×10^{-8}	2.98×10^{-3}	2.28×10^{-8}
Block 28	6.5	430	2.78×10^{-3}	3.23×10^{-8}	0.35×10^{-3}	0.41×10^{-8}
	12.6	270	2.57×10^{-3}	2.63×10^{-8}	0.49×10^{-3}	0.51×10^{-8}
	15.6	210	2.36×10^{-3}	3.13×10^{-8}	0.49×10^{-3}	0.65×10^{-8}
0.4-CM RADIUS PIEZOMETER PROBE ISHTE SIMULATION						
Piezometer #1 near field	16.9	18.5	2.19×10^{-3}	1.50×10^{-7}	2.44×10^{-3}	1.68×10^{-7}
Piezometer #2 far field	26.4	132	2.05×10^{-3}	1.69×10^{-7}	5.13×10^{-4}	4.24×10^{-8}

[a] $C_h \cong C_v$

TABLE 8—*Measured values of the coefficient of permeability* k, *coefficient of consolidation* C_h, *and shear modulus-strength ratio* G/S_u.

Site	Coefficient of Consolidation C_h, cm^2/s	Permeability k, cm/s	Shear Modulus G/S_u
	MISSISSIPPI DELTA		
1977	1.71 to 3.62 × 10^{-4}	1 to 10 × 10^{-7} [a]	.
Block 28	2.33 × 10^{-4} average[b]	. . .	. . .
1978	0.965 to 5.80 × 10^{-4}	1.11 to 14.0 × 10^{-8}	.
Block 73	2.34 × 10^{-4} average	3.89 × 10^{-8} average	29[c] to 32[d]
	ISHTE SIMULATION		
	2.8 to 17.2 × 10^{-4} [e]	2.6 to 11.6 × 10^{-7}	15.3[e]
	7.5 × 10^{-4} average	5.5 × 10^{-7}	

[a]Ref *19*.
[b]Personal communication Dr. L. Shephard, Texas A&M, to Mr. D. Lambert, NORDA.
[c]Remolded sample G_u/S'_u = 29, vane; G_{10}/S'_u = 130, simple shear [*20*].
[d]Ref *21*.
[e]G = 72.0 kPa, S_u = 4.7 kPa [*18*].

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